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NUMERICAL MODELS TO SIMULATE THE SEISMIC RESPONSE OF RC STRUCTURES

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SUMMARY

Seismic analysis of RC structures requires realistic and simple analytical models. In this paper are presented two models proposed and implemented in the program PORANL, to represent the RC buildings behaviour under severe earthquake loading. The first model represents the influence of infill walls in the global building response, and the second model represents the non-linear shear behaviour of RC elements.

The masonry infill walls are commonly used as non-structural components. It is highly recognized that the response of RC buildings to earthquake loads can be substantially affected by the influence of infill walls. The proposed numerical non-linear model for the masonry infill walls is an upgrade of the commonly used equivalent bi-diagonal compression strut model.

The response of RC elements to earthquake loads can be controlled by bending or shear behaviour, depending on the geometrical characteristics of the elements and on the reinforcement detailing. To represent the shear behaviour, in elements where shear is not negligible, it was developed and implemented in the PORANL a non-linear shear model.

Finally, in this paper are presented and discussed the results of a set of numerical calibration analyses based on tests on full-scale frames.

1. INTRODUCTION

From the observation of collapsed and severely damaged structures during recent earthquakes, it is clear the complex behaviour of RC buildings, and particularly under seismic actions. This fact underlines the need for refined numerical models that represent the behaviour of these structures at local and global levels. In this paper are presented two proposed numerical models, implemented in a structural analysis program (PORANL). The first, to simulate the participation of the masonry infill panels in the global behaviour of the RC structures, and, the second, a simplified non-linear shear model for RC elements. In the analysis of RC structures, subjected to seismic actions, the use of non-linear models (monotonic behaviour laws combined with appropriate hysteretic rules) allows to a more rigorous representation of its response [Rodrigues *et al.*, 2004].

In fact, the original version of the PORANL program was able to represent the non-linear bending behaviour of RC elements (beams and columns). Each RC structural element is modelled by a macro-element defined as the association of three bar finite elements, two with non-linear behaviour at its extremities (plastic hinges), and a central element with linear behaviour, as represented in Figure 1 [Varum, 1996]. The non-linear behaviour of the plastic hinge sub-elements is controlled through a modified hysteretic procedure, based on the Takeda model, as illustrated in Figure 2. This model developed by Costa [Costa, 1989] represents the response of a RC cross-

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section to seismic actions and contemplates typical mechanical behaviour effects as stiffness and strength degradation, pinching, slipping, internal cycles, etc.

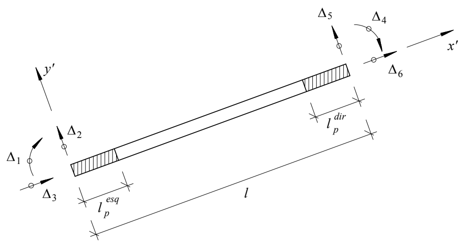


Figure 1: Frame macro-element

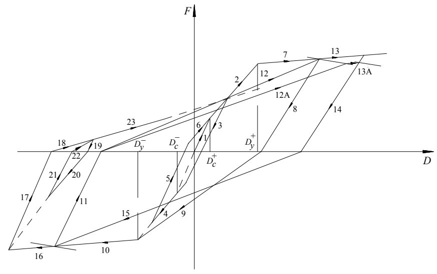


Figure 2: Hysteretic model for RC elements in bending

2. TYPICAL CAUSES OF DAMAGE AND FAILURE OF RC BUILDINGS UNDER SEISMIC LOADS

There are many reasons that justify the damages and failures of RC buildings under seismic action. The most common courses are associated with: *i*) stirrups/hoops, confinement and ductility; *ii*) bond, anchorage, lap-splices and bond splitting; *iii*) inadequate shear capacity and failure; *iv*) inadequate flexural capacity and failure; *v*) inadequate shear strength of the joints; *vi*) influence of infill masonry; *vii*) vertical and horizontal irregularities; *viii*) higher modes effect; *ix*) strong-beam weak-column mechanism, and, *x*) structural deficiencies due to architectural requirements [Varum, 2003]. However, it should be noted that some times de damages and failures are associated to a combination of these factors. In this paper, are proposed two models, one for the simulation of infill masonry panels, and other for the simulation of the non-linear shear behaviour. Next are presented some aspects relation to the seismic behaviour associated with these phenomenons.

2.1 Influence of infill masonry walls on the structural seismic response

It is inadequate to assume that masonry infill panels are always beneficial to the structural response. The contributions of the infill for the building seismic response can be positive or negative, depending on a series of phenomena and parameters as, for example, relative stiffness and strength between the frames and the masonry walls. In recent earthquakes, numerous buildings were severely damaged or collapsed due to the structural modifications of the basic structural system induced by the non-structural masonry partitions (see Figure 3).

Depending on the situations in which masonry walls extend, for example only to part of the storey-height (short-columns) leaving a relatively short portion of the columns exposed, may also induce vulnerable behaviour. Frequently, a column is shortened by elements which have not been taken into account in the global design (for example: window openings or landing slabs of staircases).

Figure 3: Column shear failure Damages on masonry infill walls

2.2 Inadequate shear behaviour

Typical gravity and wind load designs normally results in a design shear force significantly lower than the shear force that could be developed in a column during seismic loading. Therefore, shear limit states should be avoided in the seismic resistant structures. For this goal, the shear demand should be limited or shear capacity should be enhanced. The problem of shear strength and confinement is commonly more severe in corner columns, especially if the building has significant eccentricity between the centre of mass and the centre of resistance. When the load in the strong axis direction, column often fail in shear (see Figure 4-a), Another common problem is to artificially shorten a column, provoking stiffer ones, attracting much higher shear forces than they were designed to carry, short columns are vulnerable to shear failure as shown in Figure 4-b, where a column shear failure is induced by the partial infill walls is shown [Varum, 2003].

a) b)

Figure 4: Column shear failure

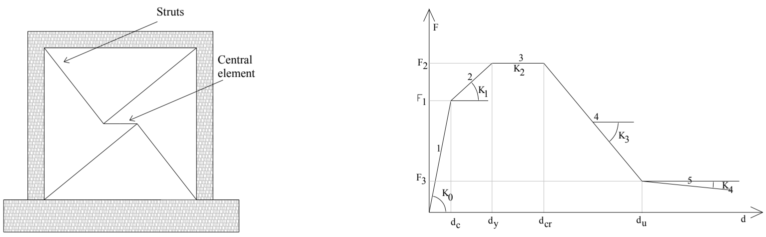
3. INFILL MASONRY MODEL

3.1 Generalities

The presence of masonry infill walls in RC buildings is very common. However, even nowadays, in the design of new buildings and in the assessment of existing ones, the infills are usually considered as non-structural elements and their influence in the structural response is ignored [Rodrigues *et al.*, 2005]. For horizontal loading, infill panels can drastically modify the response, attracting forces to parts of the structure that have not been designed to resist them [Paulay and Priestley, 1992]. Numerical analysis of RC buildings should account for the influence of infill masonry walls in their response to cyclic loading, as the produced by earthquakes.

3.2 Proposed infill masonry panel's model

The proposed macro-model is an improvement of the commonly used equivalent bi-diagonal-strut model. The proposed model considers the interaction of the masonry panel' behaviour in the two directions. Thus, the damage of the panel in one direction affects its behaviour in the opposite direction. To represent a masonry panel are considered: four support strut-elements with rigid-linear behaviour; and, a central element where the non-linear hysteretic behaviour is concentrated (see Figure 5-a). The non-linear behaviour is characterized by a multi-linear envelop curve, defined by nine parameters (Figure 5-b), representing: cracking, peak strength, stiffness decreasing after peak strength and residual strength, for each direction. The hysteretic are controlled by three additional parameters, namely: α - stiffness degradation; β - "pinching" effect; and, γ - strength degradation [Rodrigues, 2005].



a) b)

Figure 5: Implemented model for the infill masonry panel: a) Equivalent strut model; b) Global force-displacement monotonic behaviour curve

3.3 Hysteretic rules associated to the masonry infill model

The non-linear behaviour of the central element is characterized by hysteretic rules based on the Takeda's model [Takeda *et al.*, 1970], allowing to determine the response to cyclic loading depending on the material's behaviour characteristics (defined by the envelop curve and hysteretic parameters). The hysteretic rules are schematically exemplified in Figure 6. The loading stiffness depends on the maximum force and displacement values reached in previous cycles (F_{max} and d_{max}). The loading begin at the point corresponding to null-force (d_r) and its stiffness is defined according to Eq. 1:

$$K_r = \frac{F_{\max}}{d_{\max} - d_r} \tag{1}$$

The unloading is associated to a load inversion. The unloading stiffness depends on the maximum displacement reached. Before the yielding-point has been reached, the unloading stiffness (K_d) will be equal to the initial undamaged stiffness (K_0). If the maximum displacement reached is larger than the yielding displacement, but smaller than d_{cr} (cracking displacement), the unloading stiffness (K_d) will depend on the parameter α, and on the maximum displacement reached in that cycle, defined by:

$$K_d = \frac{F_{cr} - \alpha \cdot F_y}{K_0 \cdot d_{cr} + \alpha \cdot F_y} \cdot K_0 \tag{2}$$

If the maximum displacement reached is larger than d_{cr}, the unloading stiffness (K_d) will depend only on the parameter α. The unloading stiffness is then given by Eq. 3:

$$K_d = \frac{F_{cr} - \alpha \cdot F_y}{d_{cr} \cdot K_0 - \dfrac{\alpha \cdot F_y}{K_o}} \cdot K_0 \tag{3}$$

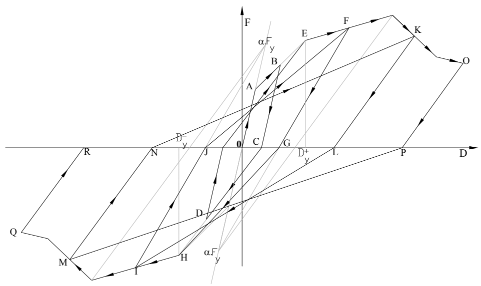


Figure 6: Hysteretic rules for the implemented masonry model

"Pinching" effect simulates the masonry cracks closing in the unloading-reloading branch. The pinching effect is represented dividing the reloading branch in two sub-branches with different stiffness (see Figure 7). The pinching effect is controlled through the parameter β, and depends on the maximum displacement reached previously.

The strength degradation, for repeated cycles of a certain displacement amplitude, was implemented considering influence of the degradation in one direction on the other.

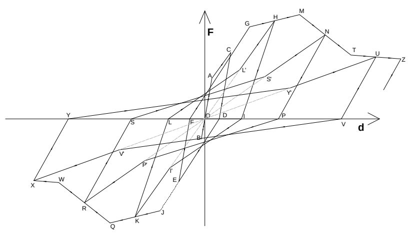


Figure 7: "Pinching" effect

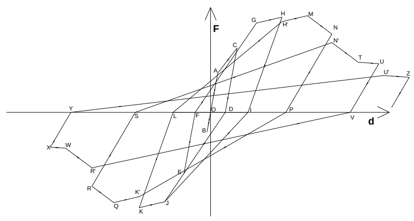


Figure 8: Strength degradation

4. MODEL FOR SHEAR BEHAVIOUR OF RC ELEMENTS

4.1 Introduction

The seismic response of slender RC structural elements is dominated by flexure behaviour. But, when the slenderness drops to a certain level, the behaviour is controlled by shear. Shear behaviour is characterised by very low ductility and, generally, by poor performance under cyclic loading. In RC building structures, it is common to use RC walls to increase the global stiffness of the structure, and therefore, controlling the deformation demands. Current analysis programs support non-linear models just in bending. A new non-linear shear behaviour model was proposed, and implemented in the PORANL computer program [Varum, 1996].

4.2 Proposed macro-model

The non-linear shear behaviour model was implemented based on the frame macro-model available in the PORANL program for bending. Therefore, each RC structural element is modelled as the association of three bar finite elements, two with non-linear behaviour at its extremities, and a central element with linear behaviour, as represented in Figure 1. The non-linear monotonic behaviour curve of an element is characterized through a tri-linear force-distortion relationship. The hysteretic rules are controlled by three additional parameters, namely: α - stiffness degradation; β - "pinching" effect; and, γ - strength degradation [Rodrigues, 2005].

4.3 Hysteretic rules

In the shear model, the non-linear behaviour is characterized by hysteretic rules based on the Takeda's model [Takeda *et al.*, 1970], allowing to determine the response to cyclic loads depending on the material's behaviour (defined by the envelop curve and hysteretic parameters). The hysteretic rules are briefly exemplified in Figure 9.

The loading stiffness depends on the maximum force and displacement value reached in the previous cycle (F_{max} and d_{max}). The loading begin at the point corresponding to null-force (d_r) and its stiffness is defined by the Eq. 4:

$$K_r = \frac{F_{\max}}{d_{\max} - d_r} \tag{4}$$

The unloading happens when a load inversion occurs. The unloading stiffness depends on the maximum displacement reached. Before the yielding-point has been reached, the unloading stiffness (K_d) will be equal to the initial stiffness (K_0). If the maximum displacement reached is larger than the yielding displacement, but smaller than d_{cr} (cracking displacement), the unloading stiffness (K_d) will depend on the parameter α, and on the maximum displacement reached in that cycle, defined by:

$$K_d = \frac{F_{cr} - \alpha \cdot F_y}{K_0 \cdot d_{cr} + \alpha \cdot F_y} \cdot K_0 \tag{5}$$

If the maximum displacement reached is larger than d_{cr}, the unloading stiffness (K_d) will depend only on the parameter α. The unloading stiffness is given by Eq. 6:

$$K_d = \frac{F_{cr} - \alpha \cdot F_y}{d_{cr} \cdot K_0 - \dfrac{\alpha \cdot F_y}{K_o}} \cdot K_0 \tag{6}$$

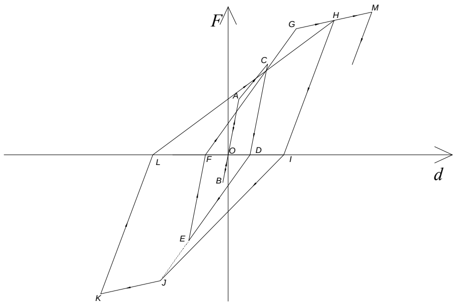


Figure 9: Hysteretic rules for the proposed shear model

The “Pinching” effect is important for elements were the shear behaviour is dominant. The pinching effect is represented dividing the reloading branch in sub-two branches with different stiffness (Figure 10). The pinching effect is controlled through the parameter β, and depends on the maximum displacement reached previously.

The strength degradation, for repeated cycles of certain distortion amplitude, was implemented considering interaction between the degradation in shear of one direction in the other.

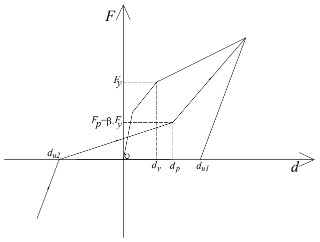


Figure 10: “Pinching” effect

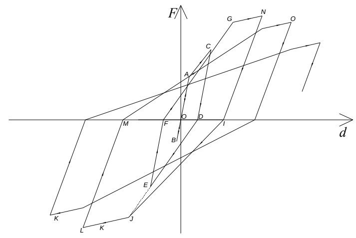


Figure 11: Strength degradation

5. CALIBRATION OF THE NUMERICAL MODELS

The proposed macro-models to simulate the influence of the infill masonry panels and the shear behaviour were verified using results obtained from a experimental campaign at the research network ICONS, addressing the seismic assessment and retrofitting of existing structures,. The experimental research work includes several studies carried out at the ELSA laboratory, at the JRC. Two full-scale four-storeys, three-bays reinforced concrete frames representative of the existent RC structures (one as a bare frame and other as a infilled frame), were designed, constructed and tested in sequence using pseudo-dynamic testing procedures. The frames were subjected separately to pseudo-dynamic tests to assess the earthquake performance of each, the bare concrete frame and the frame structure with infill masonry walls (Figure 12) [Varum, 2003].

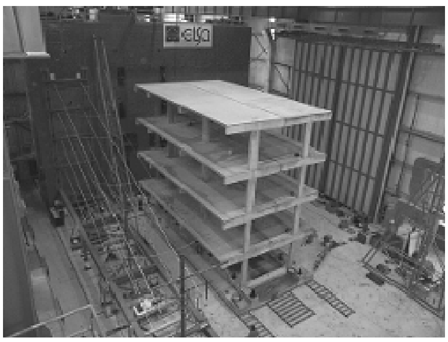


Figure 12: General view of the full-scale 4 storey RC frames

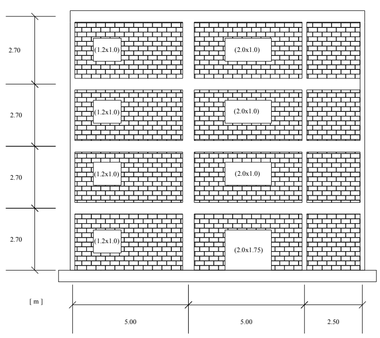


Figure 13: Frame and infill geometry

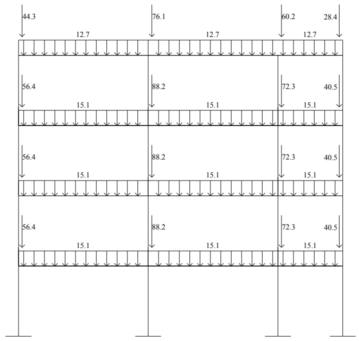


Figure 14: Vertical loads

The general layout and dimensions of the structure under analysis are given in Figure 13. The structure is represented by a plane frame model (considering three DOF's per node, i.e. two translations and one rotation) with four storeys and three bays. The cross-sections' geometrical characteristics and the reinforcement detailing of the columns and beams as well as the infill masonry properties can be found in the literature [Varum, 2003].

5.1 Calibration of infill masonry model

Using the numerical models implemented in the PORANL program, namely the non-linear bending behaviour of RC frames elements, by Varum [Varum, 2003], and the non-linear infill model presented in present sections, the non-linear response of the RC frames was computed for each earthquake input motion applied in the pseudo-dynamic tests.

Earthquakes of 15 seconds duration and increasing return periods, corresponding to 475, 975 and 2000-yrp, were applied to the frames. During the PsD tests, collapse was verified at 5.0 seconds of the 2000-yrp earthquake for the infilled frame. Therefore, the numerical analysis was performed for the earthquake input motion series presented in Figure 15.

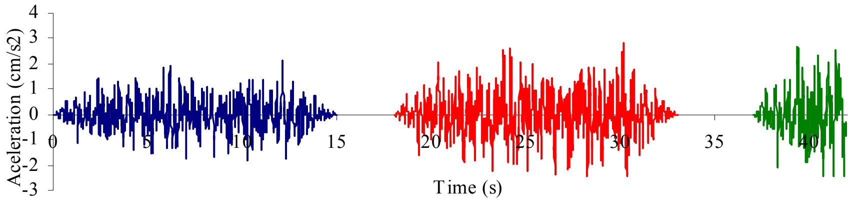


Figure 15: Earthquake input motion applied in the numerical analysis

Results in terms of storey displacements time histories (experimental PsD tests and numerical results) are presented in Figure 16. Figure 17 shows storey shear versus inter-storey drift curves at storey level. Figure 18 shows the evolution of the dissipated energy at each storey level.

The numerical results confirm that the models adopted (RC elements with non-linearity concentrated at their extremities and the proposed macro-model for the infill panels) are adequate to simulate the non-linear seismic response of infilled masonry RC frames. The masonry numerical model was able to represent the structural protection by infill masonry panels, for low intensity seismic actions, and to predict the infill masonry walls failure. In fact, the soft-storey mechanism is well identified with the numerical model proposed.

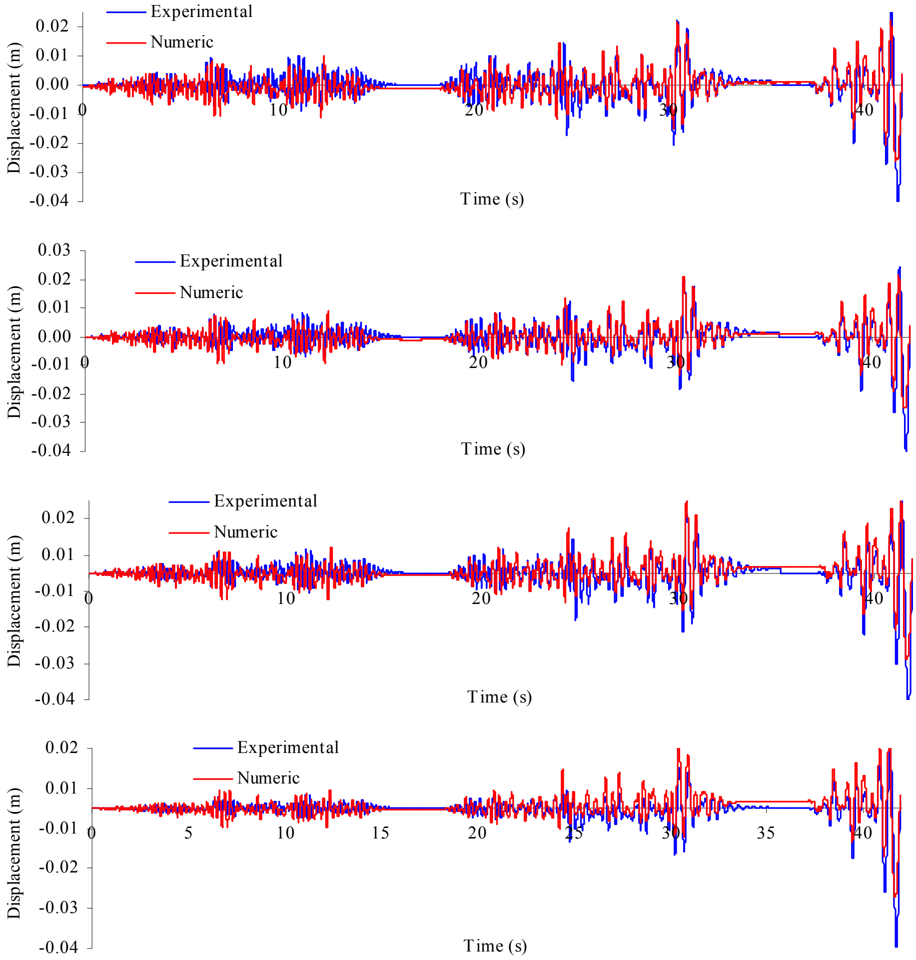
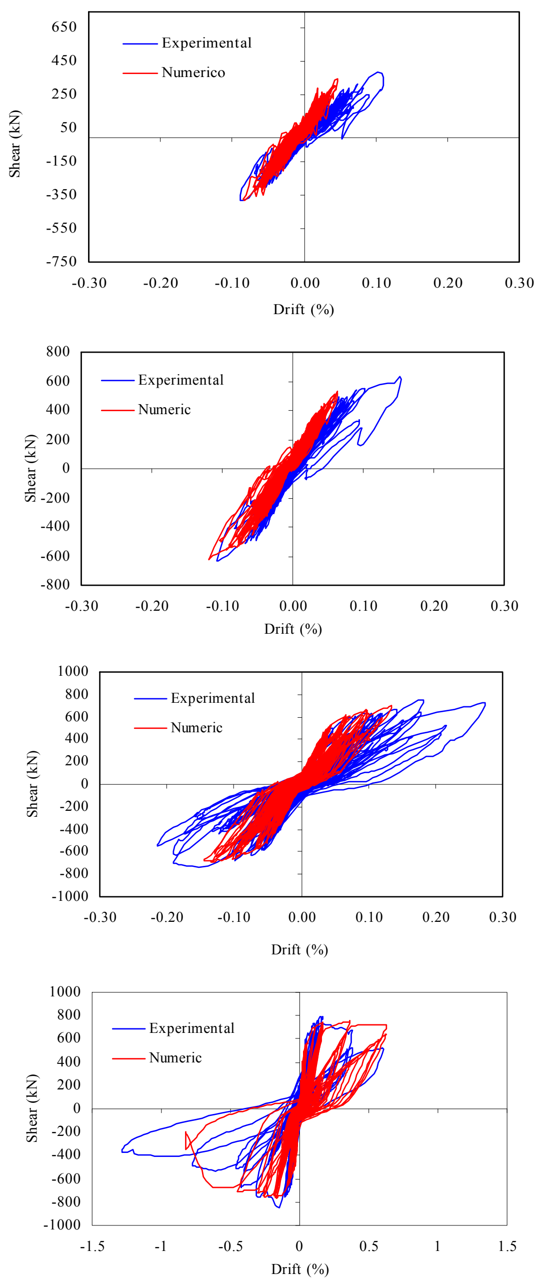


Figure 16: Storey displacements (4th, 3rd, 2nd, 1st)



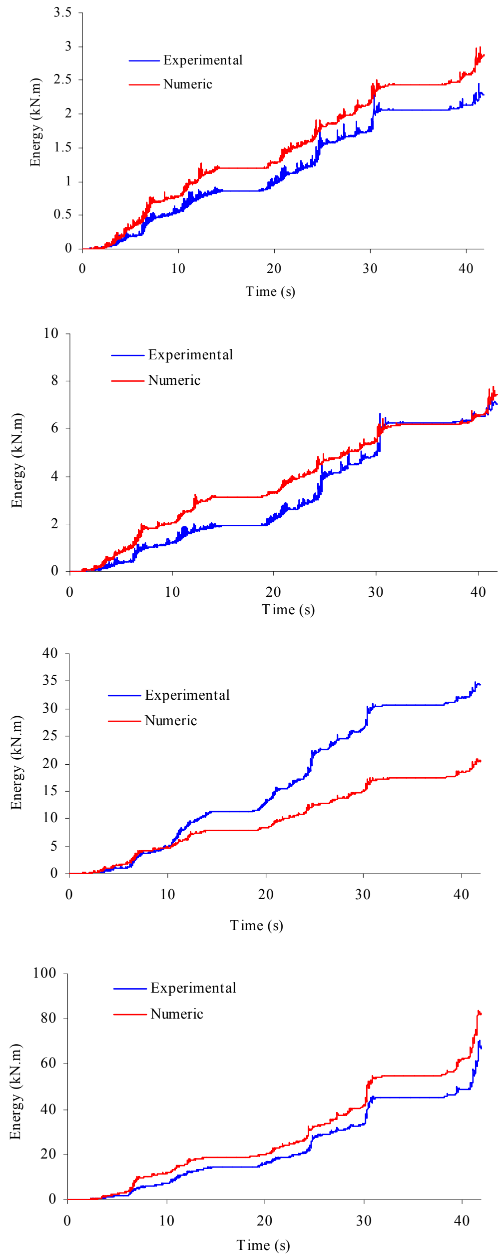


Figure 17: Base-shear versus inter-storey drift (4^{th}, 3^{rd}, 2^{nd}, 1^{st})

Figure 18: Total energy dissipation evolution (4^{th}, 3^{rd}, 2^{nd}, 1^{st})

5.2 Calibration of the non-linear shear model

To illustrate the ability of the proposed shear model, it were simulated the response of the first storey strong column of the ICONS frame [Varum, 2003]. The studied column was exhaustively instrumented (see Figure 19 and 20-a). For the tests, were installed in the column, a set of 27 relative displacement transducers, located as represented in Figure 19, witch allows to capture the column deformation (in bending and in shear) at three levels. To reproduce the measured deformations during the tests, it was build a simplified model for the column. The column was simulated with the following boundary conditions (Figure 20-b): a) displacements and rotations blocked at the base; b) compression axial force was applied, corresponding to the vertical loading; c) imposed lateral displacement (Figure 21) and rotation (Figure 22) at the top of the column, according to the measured results (local instrumentation) during the tests. For the imposed conditions, it was performed two series of analysis. First, it was considered only the bending behaviour. Secondly, it was considered the bending and shear behaviour.

From the experimental results, the shear force attracted to the strong-column was estimated as a parcel of the total storey shear with simplified processes [Rodrigues, 2005].

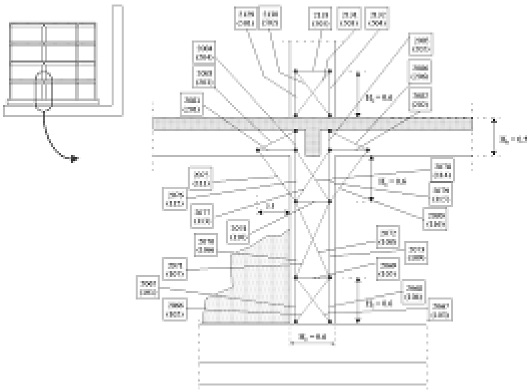


Figure 19: Instrumentations set-up at the 1st storey strong-column

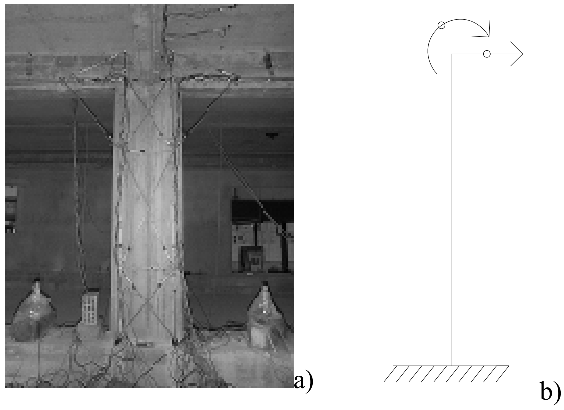


Figure 20: Studied column: a) The storey instrumentation; b) Model studied

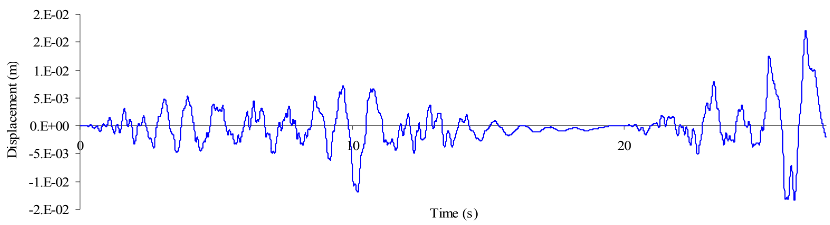


Figure 21: Horizontal displacement imposed at the top of the column

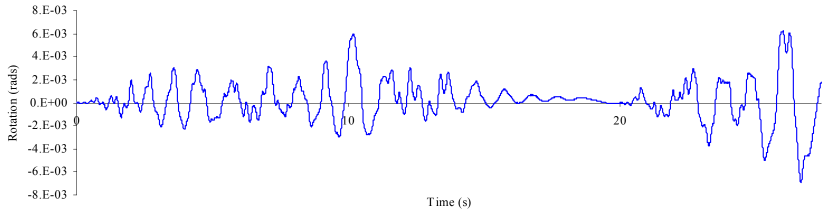
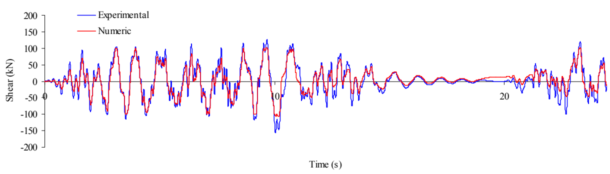


Figure 22: Rotations imposed at the top of the column

In the first analysis was modelled only the bending behaviour. In Figure 23 is presented the evolution of the shear force in the column (numerical and experimental results). In Figure 24 are represented the column shear-drift results.



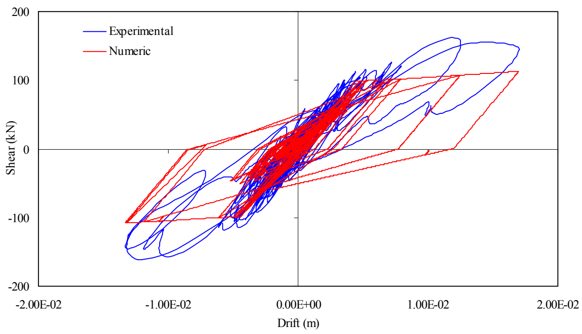


Figure 23: Column shear evolution considering only bending behaviour

Figure 24: Shear-Top-Displacement, considering only bending behaviour

For the second analysis, it was modelled the response of the strong column to the imposed boundary conditions, considering the shear and bending behaviour. Figure 25 is presented the evolution of the shear force in the column (numerical and experimental results). In Figure 26 are represented the column shear-drift results.

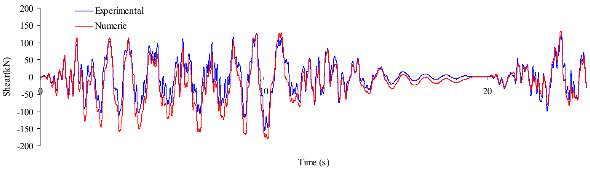


Figure 25: Column shear evolution in time considering bending and shear behaviour

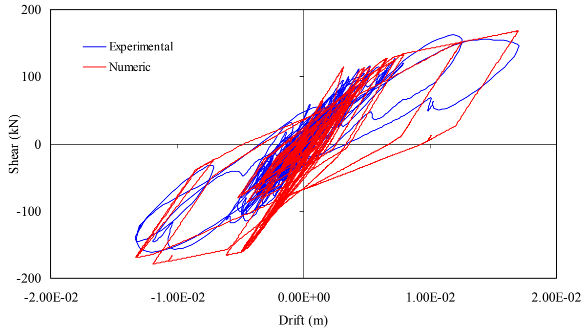


Figure 26: Shear-Top-Displacement, considering bending and shear behaviour

From the results obtained, it can be concluded that for RC elements with considerable shear stiffness, the bending behaviour may not be able to accurately reproduce the behaviour under cyclic loading, particularly for high demand levels. The numerical results presented illustrates that the combination of the bending and shear behaviour provides a better representation of the experimental results.

6. FINAL REMARKS

Structural analysis programs that include non-linear models are valuable tools in the analysis and verification of structural safety, giving the engineer capacity to represent more precisely the real behaviour of the structures. For design of new structures or capacity assessment of existing ones, nonlinear analyses allow for a better representation of the structural response under any loading condition, and under earthquake loading in particular.

The proposed models were able to reproduce well the experimental results, not only in terns of the maximum peaks values but also in terms the dissipated energy and hysteretic behaviour, however a more exhaustive testing campaign would help to calibrate the proposed models. The program is now able to represent the influence of infill masonry panels in the global response of the structure and also to take in account the shear behaviour in RC elements, which will permit a future exhaustive analysis campaign that would help to understand the behaviour RC building under earthquake loads.

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